PISTORIUS

Design of a Steel Skeleton Office Building

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DESIGN OF A STEEL SKELETON OFFICE BUILDING

BY

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I recommend that the thesis prepared under my supervision by BERNHARD HENRY PISTORIUS entitled Design of a Steel Skeleton Office Building be approved as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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DESIGN OF A STEEL SKELETON OFFICE BUILDING

INTRODUCTION

There are four general types of construction applicable to tall buildings, as follows:

(1) Buildings in which the exterior walls are designed to carry their own weight, together with the wall ends of the girders which carry the floor loads. The interior floor loads are carried by beams and girders to steel or cast-iron columns.

(2) Buildings in which the exterior walls carry only their own weight, the wall ends of the floor girders and beams being carried by the columns.

(3) Buildings in which the exterior walls and floors are all carried by a steel framework of columns, girders, and beams.

(4) Buildings constructed of reinforced concrete.

Structures of the first class are limited in height to about ten or twelve stories on account of the excessive thickness of walls at the base and the large bearing loads upon the soil.

The second type of construction permits a lighter exterior wall than the first. It also has an advantage in that all the columns are carried on isolated piers, and a more equal settlement can be obtained. Furthermore, the piers carrying the walls are independent of those carrying the floor

loads, therefore an unequal settlement of one pier with respect to another will cause no damage either to the walls or to the steelwork.

The third type, known as the "cage" or "skeleton" construction, is the one used in the construction of modern skyscrapers. It permits the use of thin exterior walls, thus reducing their weight to a minimum. The foundations consist of an isolated pier under each column, which can be proportioned so that the settlements will be nearly equal. The use of cantilever footings, which is very often necessary when it is desired to keep all foundations inside of the building line, is more easily applied to this type. The "cage" construction permits of great rapidity in erection. Often the entire steel work, floors, and exterior walls of large skyscrapers are completed in from three to six months. The use of terra-cotta as a fire-proofing material is a common practice, and often exterior walls, floors, and partitions are all built of this material. Concrete has been used to a considerable extent in floor construction.

Tall reinforced-concrete buildings are built using a construction similar to the steel skeleton type. The columns, girders, and beams are built of reinforced concrete and the floors are reinforced-concrete slabs, which are carried by the beams and girders. In some forms of construction the beams and girders are omitted, the floors consisting simply of a reinforced-concrete slab, supported by the columns. The exterior walls are sometimes built of concrete, but are more often constructed of brick or terra-cotta which,

as in the case of the steel skeleton type, is supported at each floor by beams. Buildings of this type are generally built to moderate heights, only; however, some twelve-story structures have been erected.

In taking the design of a steel skeleton office building for a thesis, it will be necessary to limit the extent to which the various parts of the design are developed, due to the great variety of the different branches of construction involved, and to the great number of details. It will be endeavored in this design to treat of only that portion of the design of the superstructure which is considered the work of the civil or structural engineer. The architectural features will be considered only in a general way, merely for the purpose of furnishing a working basis for computing the loads carried by the steel framework. In the design of the framework only the sizes and shapes of the different members will be determined, although the method of framing will be shown. No shop details or working drawings will be made. The design of the plumbing, heating, and ventilating will not be considered.

The design will be taken in the following order:

Architectural Features.

Loads and Stresses.

Floor Systems.

Columns.

Spandrels and Cornices.

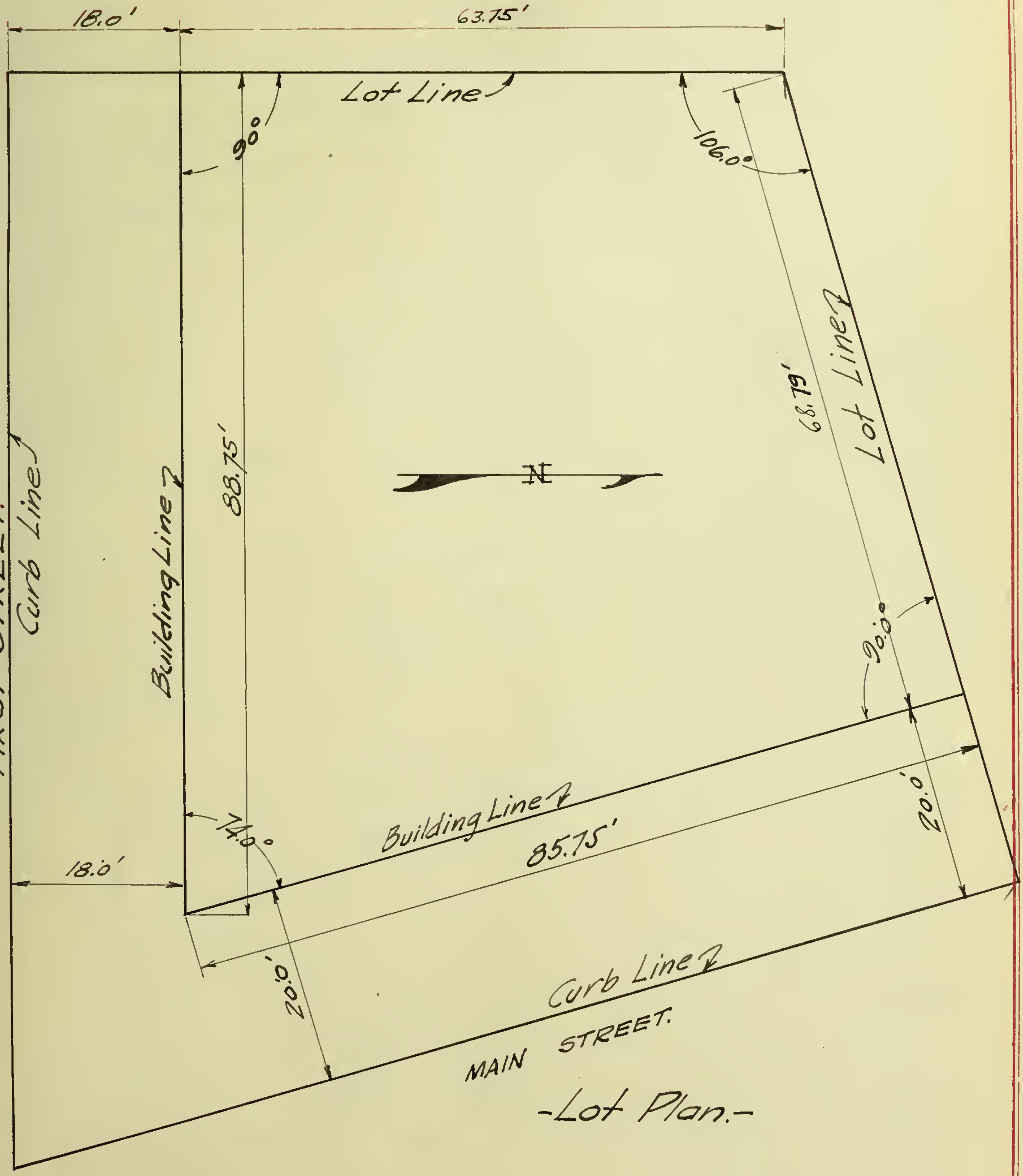
Wind Bracing.

ARCHITECTURAL FEATURES

The design of a modern office building depends upon so many factors that it will be necessary to make some assumptions for the building under consideration. The building designed will be taken as twelve stories in height. The ground floor will be used for stores, and the eleven floors above will be used for business offices. Plate (I) shows the lot-plan. The lot is a typical corner one at an oblique street intersection, its dimensions and shape having been chosen at random. It is assumed that the building is located in Chicago. C. C. Schneider's "Building Specifications" will be used.

In office buildings the offices are generally arranged to suit the tenant. It is necessary, however, for the architect to determine the location of light courts, stairs, elevators, and corridors, so that he may get the most convenient and economical arrangement for the offices. The main entrance of the building should be on the main-street side of the building. The elevators should be centrally located and should be close to the entrance, so that there will be as little loss of floor-space as possible. In the floor-plan layout the location of columns should be given much consideration. The ideal distribution of columns would be that in which all the framing would be rectangular, all spans nearly equal and of such dimensions that the beams and girders would be stressed to just the working limit. This distribution of

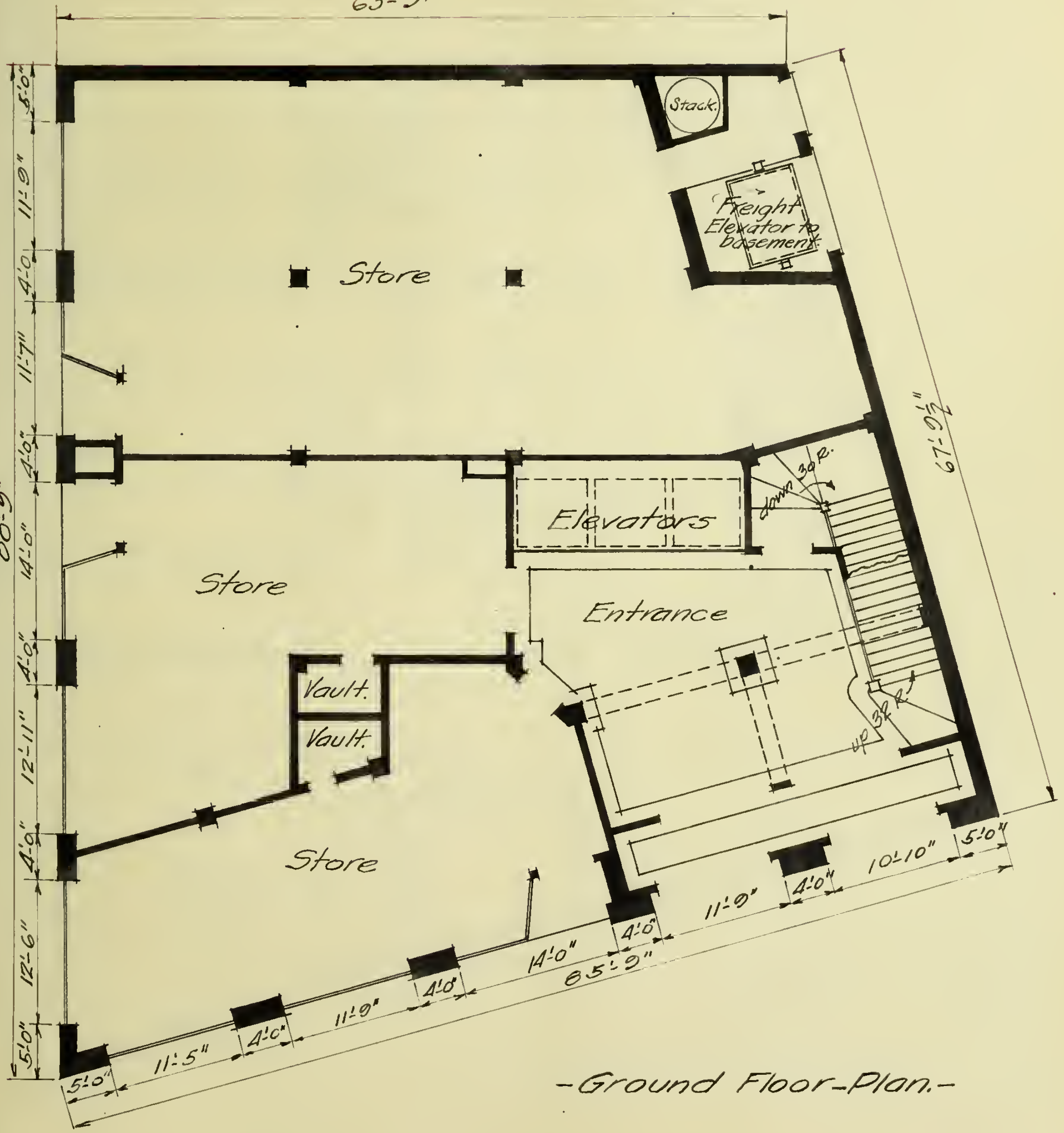
PLATE-I.



-Lot Plan.-

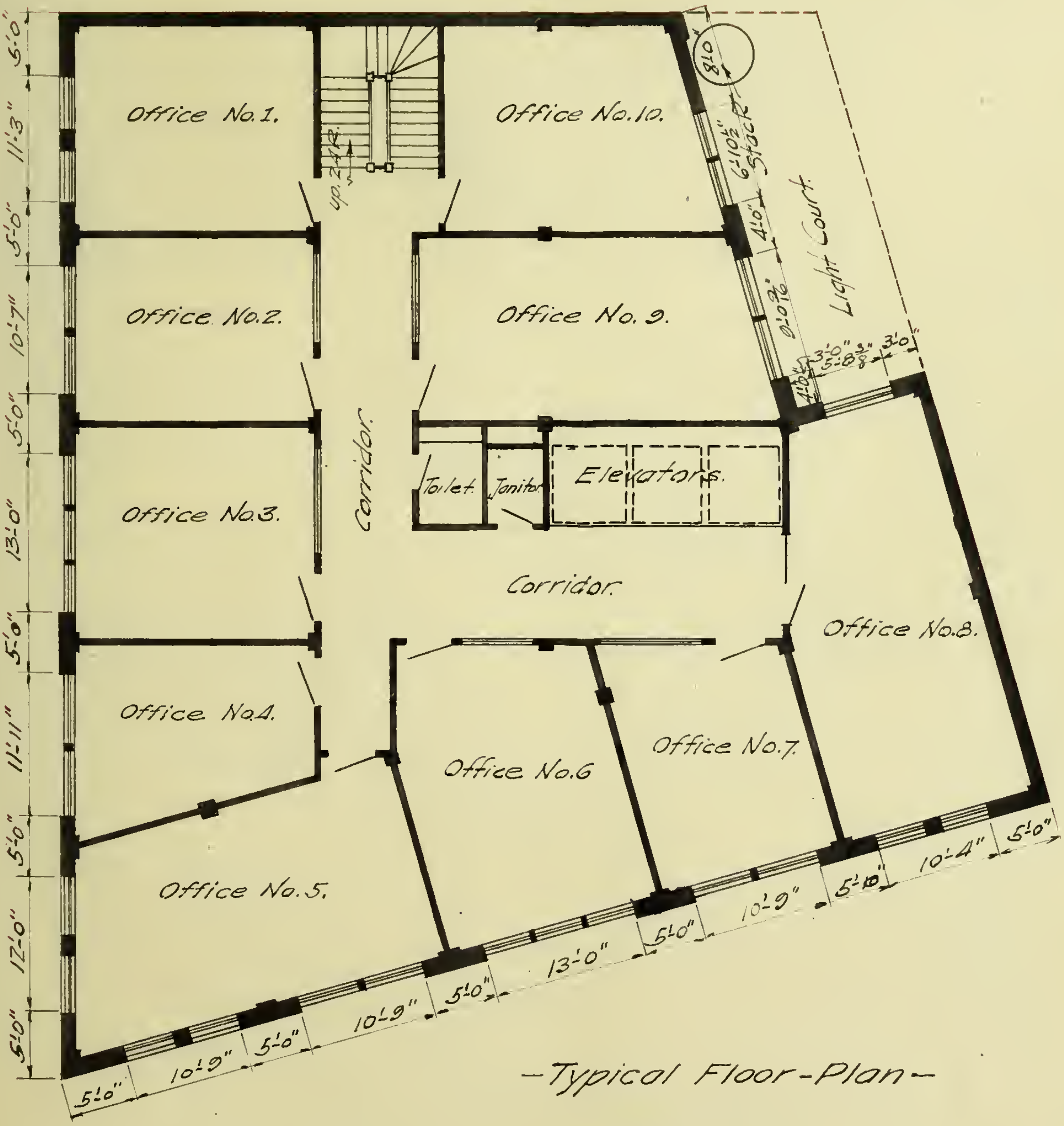
columns can very rarely be obtained in office buildings, because the location of the columns will, in most cases, be governed by the arrangement of the office and corridor partitions and other architectural features. It is a general practice to place columns at the intersections of corridor and office partitions. In this building, however, the above arrangement would prove impracticable as it would necessitate additional columns or long girder spans, which would be more expensive than if one row of columns be set back from the corridors and run through the partitions between offices. It will be necessary to try numerous column-layouts and floor-plans for any particular lot before the best arrangement can be determined. The floor-plans used for this design are the result of several trials of possible layouts, and in the writer's judgment are the best that could be devised for this particular lot. Plates (II), (III), (IV), and (V) show the plans for the ground floor, second floor, typical office floor, and roof.

A light-court will be necessary as it is impossible to light all offices from the street sides of the building. In placing the light-court on the lot-line it is assumed that the adjacent property owner will build a similiar court adjoining this one. The benefits thus derived will be greater and will be obtained at a smaller loss of lot area than if the buildings were built with individual courts, which did not adjoin. The corridors are lighted by the use of sash doors and sashes placed in the corridor partitions. It will be necessary to light the stair-well by means of

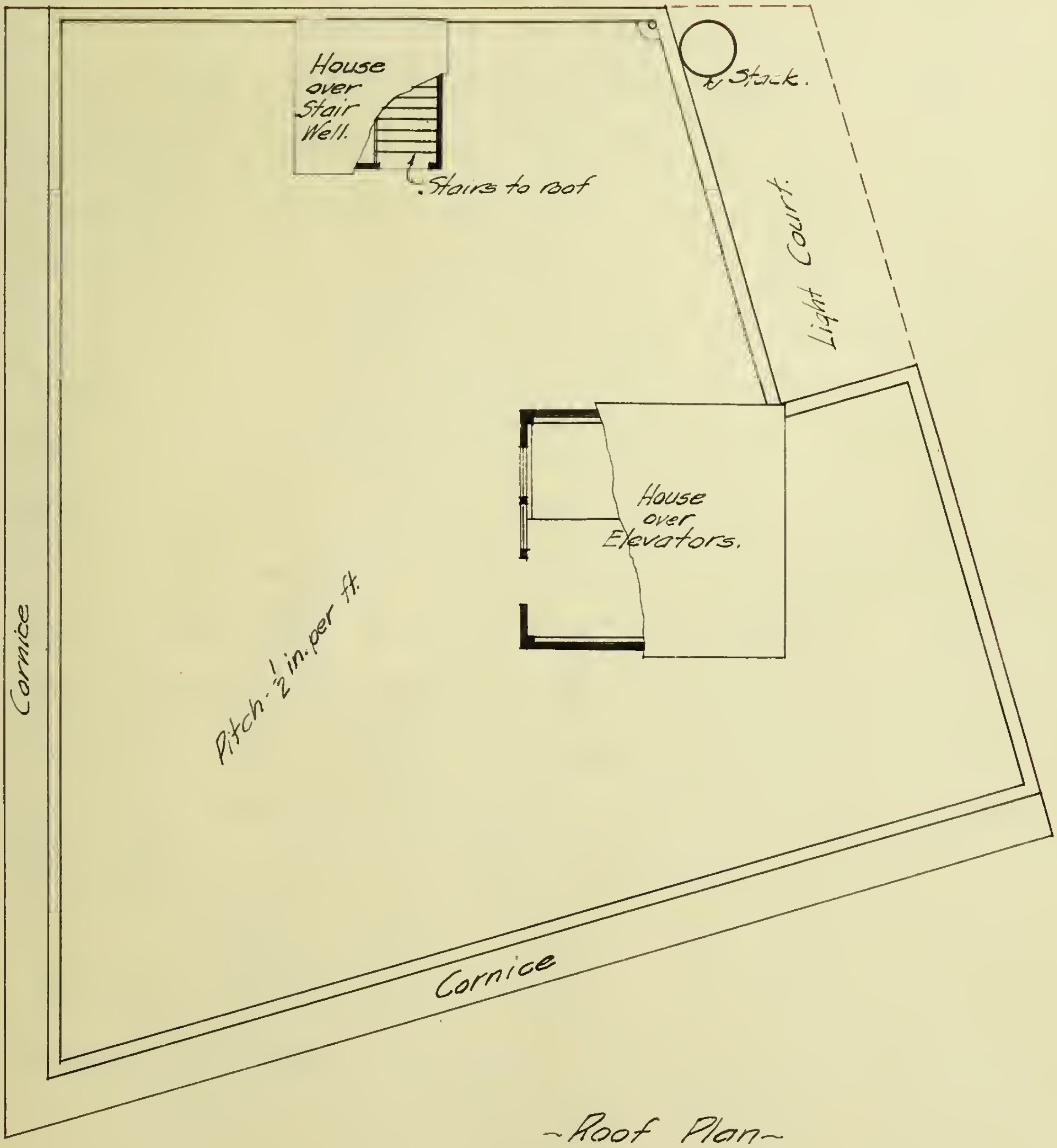


-Ground Floor-Plan.-

-Second Floor-Plan-



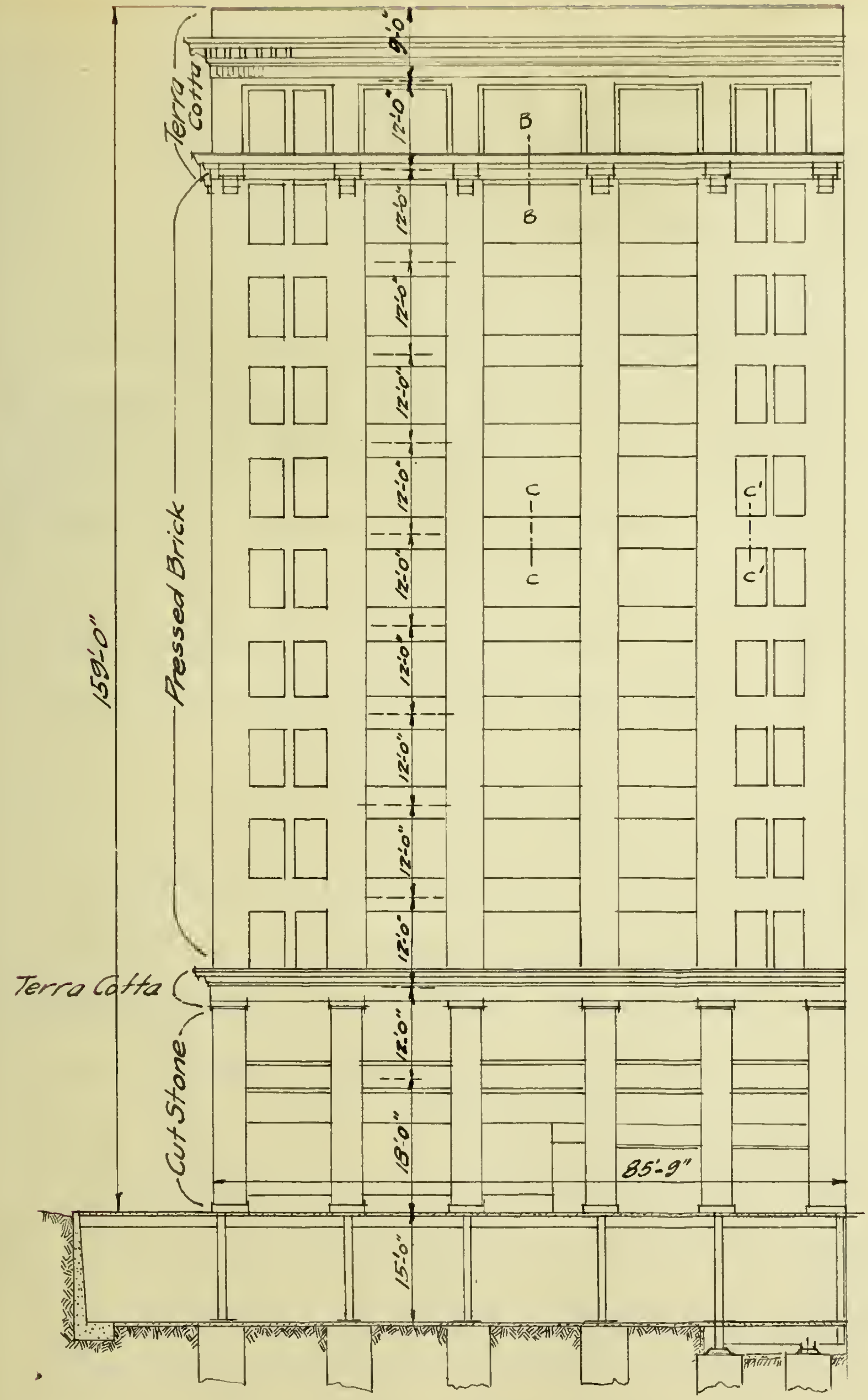
-Typical Floor-Plan-

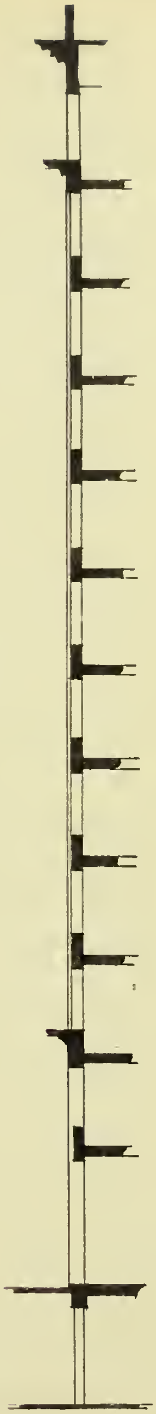

House
over
Stair
Well.
Stairs to roof
Stack.
Light Court.
House
over
Elevators.
Pitch $\frac{1}{2}$ in. per ft.
Cornice
Cornice

~Roof Plan~

artificial light. Stairs are required by the building ordinance of Chicago. This ordinance requires, for a building of this type and size, one flight of stairs at least five feet in width. Three elevators have been provided, and these should prove ample for a building of this size. These elevators will be built strong enough to carry ordinary office furniture; therefore, no freight elevator has been provided. The stack is placed in the light court instead of in the interior of the building to save floor space. Toilet facilities will be provided by two large toilet rooms, one on the fifth and the other on the seventh floor. The former is for men and the latter for women. One small toilet room for men's use, fitted with a closet and a lavatory, will be located off the corridor on each floor. In addition to these, each office will be provided with a lavatory. Other details, which must be considered in the floor-plans, are the location of vents, pipe-spaces, and vaults. The latter can be built in almost any desired location and for office purposes they are usually built of terra-cotta partition-blocks.

The elevation of the building is strictly an architectural feature and is generally decided entirely by the architect. That the steel skeleton type of building permits of a very beautiful architectural treatment is proven by numerous examples of very handsome buildings in our large cities. Much can be said concerning the aesthetic design of the exteriors of skyscrapers, but this is in the realm of the architect, not of the civil engineer. Plate (VI) shows the elevation of the building under design.





Section

-Main Street Elevation.-
(First Street Elevation will be similar.)

LOADS AND STRESSES

The loads which are considered in the design of a tall building are of three kinds; dead, live, and wind loads. The dead load consists of the weight of all the materials of construction, such as floors, beams, girders, columns, and all permanent fixtures. The live load consists of the weight of persons, office furniture, stores, moveable partitions, and moveable goods. The wind load is the pressure on the exterior of the building due to the wind. In all cases the dead loads can be determined by calculating the weights of the materials. The amount of live load per square foot of floor area depends upon the use of the floor. There is a considerable difference in the building laws of various cities as to the amount of live load to be considered in office-floor design. New York, for example, requires 100 pounds and Chicago 70 pounds per square foot of floor surface for the floor beams. A reduction of this amount in varying degrees is permitted for girders and columns; since there is a small probability that all the area supported by a girder or column will be loaded with the full live load at any time. The reduction in the live load for girders is generally 10 or 15 percent. A reduction of 15 percent, as recommended by Schneider's Specifications, will be used in this design. The New York law requires that the total live load be considered as coming upon the columns in the upper two floors, and that it shall be decreased 5 percent for each floor downward, until

a reduction of 50 percent is reached. This reduction is practically the same as that given in Schneider's Specifications and will be used in this design. Table A shows the results of this reduction in pounds per square foot for each floor. A live load of 70 pounds per square foot will be used in accordance with the Chicago Ordinance for the design of the office floors, 100 pounds per square foot for the ground floor, and 40 pounds per square foot for the roof. In the design of the framing which supports the elevators the load will be considered as doubled to take care of shocks. A load of 200 pounds per square foot will be used as live load in the design of the sidewalks framing. This amount is required by the Chicago Ordinance.

The following summarization gives the working stresses and unit dead loads, together with a table of unit dead and live loads for the columns.

ALLOWABLE STRESSES

I-beams (floor beams, girders, spandrels) 16 000 lbs. per sq. in.

Columns (concentric loading) $16\,000 - 70\,\frac{l}{r}$ lbs. per sq. in.

Columns (eccentric loading) $16\,000 - 70\,\frac{l}{r} - \frac{3}{4}\,\frac{Mc}{I}$ lbs. per sq. in.

In the above formulas,

l = unsupported length in inches,

r = least radius of gyration in inches,

c = distance to extreme fiber in inches,

I = moment of inertia in inches, and

TABLE A

UNIT COLUMN LOADS

Story	Dead Load. D	Live Load. L	Total L.Load A	Percentage for Cols. B	Total L. Col. Load C	L.Load for Cols. L′	Use of Floor.
Roof.	50	40	40	100	40	40	
12th.	85	70	110	100	110	70	office
11th.	85	70	180	95	171	61	"
10th.	85	70	250	90	225	54	"
9th.	85	70	320	85	272	47	"
8th.	85	70	390	80	312	40	"
7th.	85	70	460	75	345	33	"
6th.	85	70	530	70	371	26	"
5th.	85	70	600	65	390	19	"
4th.	85	70	670	60	402	12	"
3rd.	85	70	740	55	408	6	"
2nd.	85	70	810	50	410	2	"
1st.	95	100	910	50	455	45	Stores.
B'm't.							
Total.		910					

D - Dead load - lbs. per sq. ft.

L - Live floor beam load - lbs. per sq. ft.

A - Total of live load - lbs. per sq. ft.

B - Percentage of full live load for columns

C - Total live column load per sq. ft. of area supported.

L′ - Live column load per sq. ft for each floor.

M = bending moment in pound-inches.

The following weights of materials were used in calculating the dead load.

Steel _ _ _ _ _ _ _ _ _ _ _ _ _ _ 490 lbs./cu. ft.

Maple _ _ _ _ _ _ _ _ _ _ _ _ _ _ 3.5 lbs/board-ft.

Cinder concrete _ _ _ _ _ _ _ _ 60 lbs./cu. ft.

Terra-cotta _ _ _ _ _ _ _ _ _ _ _ 112 lbs./cu. ft.

Brick masonry _ _ _ _ _ _ _ _ _ 115 lbs./ cu. ft.

Plaster _ _ _ _ _ _ _ _ _ _ _ _ 120 lbs./ cu. ft.

FLOOR SYSTEMS

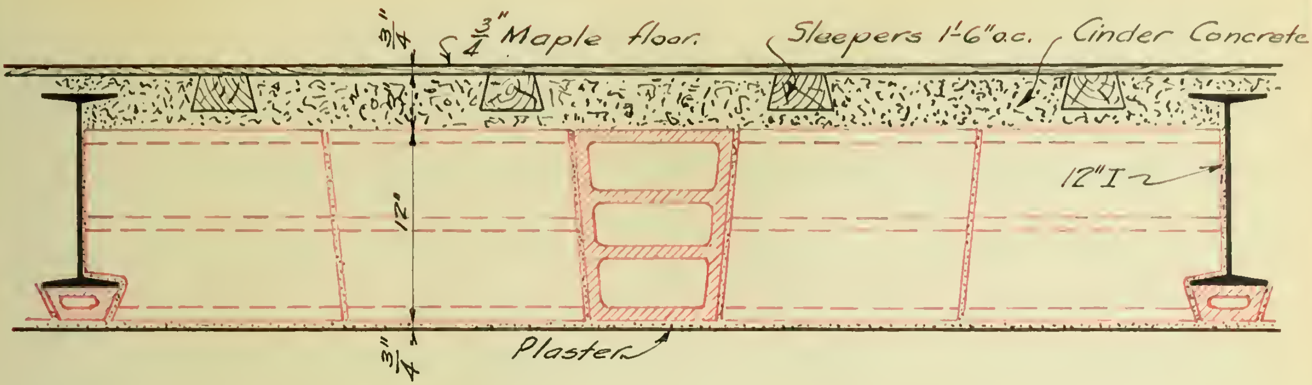
There are a great number of types of floor construction which are applicable to steel skeleton structures. The types most generally used can be divided into three classes· the hollow terra-cotta tile arch; the reinforced-concrete slab; and the combination of terra-cotta tile blocks and reinforced concrete. The hollow terra-cotta-tile arch is the most common type, and at present is used more than any other method of floor construction in steel skeleton buildings. The tremendous popularity of the steel frame construction is in a large measure due to the adaptability of hollow terra-cotta tiles in the construction of fire-proof floors and in fireproofing the steel work in general. Space does not permit an exhaustive discussion on the merits of the various types of floor-construction, but a brief description of the terra-cotta arch and its advantages will be given.

The terra-cotta type of arch is used for spans varying from about four to eighteen feet. In the smaller spans - those up to about eight feet - the flat arch is generally used, and in the larger spans the segmental arch, or cambered arch, is used. This type of floor has been proved by practical and experimental tests to be as fireproof and to afford as much protection to the steel frame as any floor system yet devised. It is recognized as "fireproof construction" by fire underwriters throughout the country. Its chief advantage over other types of construction is the rapidity with which it can be laid. The centering may be removed within twenty-four hours after the arches are laid, while for concrete floors the centering must, under the most favorable conditions, remain in place at least ninety-six hours, and often two weeks under unfavorable weather conditions. The flat arch is very commonly used owing to the ease with which it can be plastered on the underside. A flat ceiling is thus obtained without the use of a suspended framework such as would be required with a concrete floor. The terra-cotta arch also acts as a lateral bracing for the structure, since it extends the full depth of the floor beams. Concrete floors, on the other hand, cover only the upper third of the floor beams. The hollow tile arch-blocks are excellent non-conductors of sound, a very important consideration in office and hotel buildings. The concrete floor is particularly adapted to buildings of irregular shape, in which there is considerable irregular framing, since the pouring of the con-

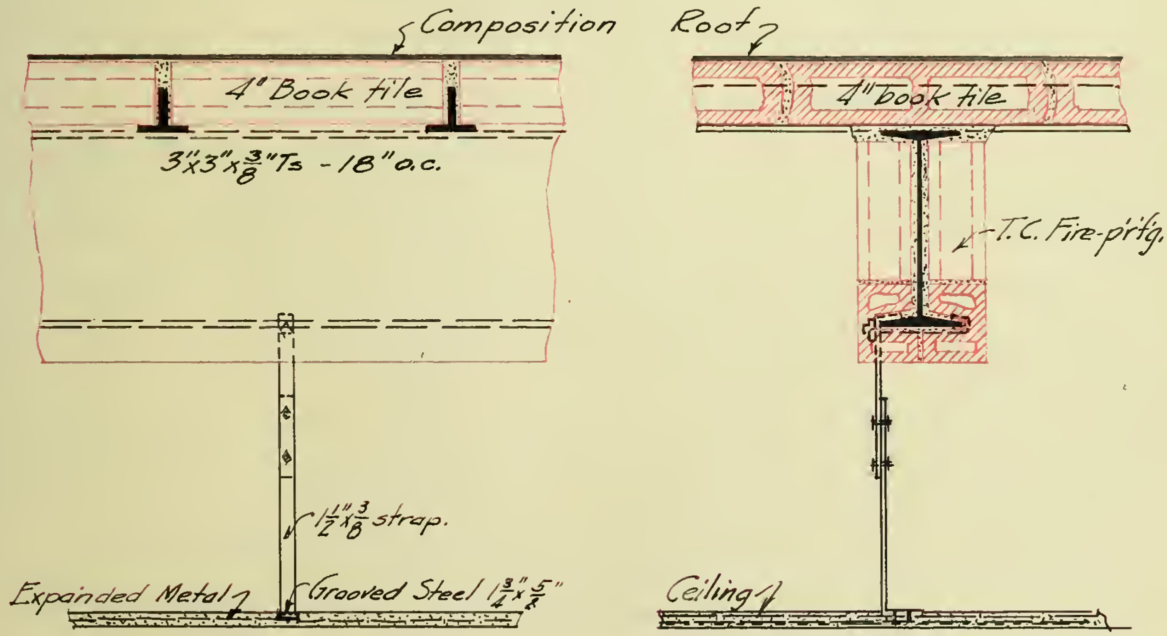
crete is not rendered any more difficult by such irregularities. In the case of terra-cotta arches, a considerable number of "special" blocks would be required under these conditions.

For a given superimposed load, the depth of arch will depend upon the span. In this building the girder spans vary from fifteen to eighteen feet and could be divided into four arch spans, varying from three feet-nine inches to four feet-six inches, or into three arch-spans, varying from five feet to six feet. With the smaller arch-span a six-inch arch, weighing about twenty-five pounds per square foot of floor and requiring floor beams ten inches in depth, could be used. As a flat ceiling without beams is desired, this depth of arch could not be used unless a great amount of filling was used to bring the floor above the top of the floor beams. This would make the floor nearly as heavy as a twelve-inch arch, and even then it would have less strength and would require more steel. This is also true of the eight-, nine-, and ten-inch arches. A twelve-inch arch will require twelve-inch floor-beams, and it can be laid with a maximum span of about seven feet. For this load and for the floor-beam spans used in this building a twelve-inch I-beam is required; therefore, a twelve-inch arch will be used in this design. Manufacturers list the twelve-inch arch, having a six-foot span, as being able to carry a superimposed load of two hundred ninety-five pounds, with a factor of safety of seven. This is amply strong for the purposes of this design. Plate (VII) shows the details of the typical floor and roof construction.

PLATE·VII.



Section thru Floor.
Showing typical floor construction.



Section thru Roof.
Showing roof construction.
and suspended ceiling.

The girders in the ground floor and second floor will consist of single I-beams of greater depth than the floor-beams, as a "beamed" ceiling in the basement and ground floor is not an objectionable feature. In the other office floors all the girders, except those which occur in the partitions, will be made up of two I-beams - properly held together by separators - of the same depth as the floor beams. This will avoid any projecting beams in the ceiling in the offices or corridors. In the design of the beams and girders around the stair-well the same dead and live load per square foot was considered for the stairs as for the floors.

Table (B) shows the computations in tabular form for the floor beams of typical office floors. As the spans and framing are similar for all floors and roof, the section moduli required for beams in both the main floor and roof will be proportional to the loads carried. These computations have not been shown. Table (C) and page 22 give the computations for the girders for the typical office floors. The computations on pages 23&24 do not include all of the girders, but are illustrative of the methods of computing beams with unsymmetrical concentrated loads. The girders of the main floor and roof were computed by proportion, as explained above. Plate (VIII) shows the general arrangement of the caissons and the cantilever girders of the foundation. Plates (IX), (X), and (XI) show the steel framing plans for ground floor, typical office floor, and roof, and give the sizes and weights of all members.

TABLE. B.

DESIGN OF FLOOR BEAMS. FOR OFFICE FLOORS.

Dead Load - 85; live load - 70; total - 155 lbs. per sq. ft.

Ref. No.	Span of Beam. ft. S.	Span of Arch. ft. D	Total Load S x D x 155# lbs. W	Bending M. W x S x $\frac{1}{8}$ lb. ft. M.	Section Modulus M x 12 ÷ 16000 in.^{4} $\frac{I}{c}$	Least Depth Section Size	$\frac{I}{c}$	Section Used Size	$\frac{I}{c}$
A 1	20	5.83	18100	45200	33.9	12" I 31.5#	36.0	12" I 31.5	36.0
2	"	5.67	17600					"	"
3	"	5.25	16300	40700	30.3	10" I 40.0	31.7	"	"
4	"	5.58	17300					"	"
5	"	6.00	18600	46500	34.0	12" I 31.5	36.0	"	"
6	"	"	"					"	"
7	"	"	"					"	"
Ba	9	5.83	8120	9130	6.85	6" I 12.5	7.3	12" I 25	36.0
2	19	5.67	16700					"	"
3	19	5.25	15450	36600	27.40	10" I 35.0	29.3	"	"
4	19	5.58						"	"
5	19	6.00	17700	42100	31.60	10" I 40	31.7	"	"
6	9	6.60	9200	10300	7.72	7" I 15	10.4	"	"
C 1	12.0	5.83	10850	16200	12.12	8" I 18	14.2	12" I 31.5	36.0
2	13.5	5.67	11850					"	"
3	15.6	5.25	12450					"	"
4	17.0	5.58	14700	31200	23.40	10" I 25	24.4	"	"
5	18.5	(2.58)	7640					"	"
6	20	(2.50)	8000					"	"
7	"	5.0	15500	38800	28.9	10" I 35	29.3	"	"
8	"	5.0	"						
D 1	17.3	5.3	14200	30700	23.1	10" I 25.0	24.4	12" I 31.5	36.0
2	"	"						"	
3	"	"						"	
4	"	5.25						"	
5	"	5.63	15100	32,600	24.5	"	"	"	
6	"	6.0	16100	34800	26.1	"	"	"	
7	"	5.63						"	
8	"	5.25						"	
9	"	5.37						"	
10	"	5.63						"	
11	"	5.5						"	
12.	"	5.0	13400	29000	21.8	"	"	"	

TABLE-C.

DESIGN OF FLOOR GIRDERS/ FOR OFFICE FLOORS.

Dead load=85; live load 70x0.85=60; total=145 lbs./sq.ft.

Ref No.	L. ft.	S ft.	D ft.	A 5xD sq.ft.	Reaction R Ax145 lbs.	Moment M $R \times \frac{L}{3}$ lb.ft.	$\frac{I}{c}$ $M \times 12 \div 16000$ in⁴	Least Depth Section.	Section Used.	$\frac{I}{c}$ in⁴
Girders with two equal conctrated loads at third points.										
G2	5.58	19.5	5.25	102.2	14800	76900	57.6	15"I42#	15"I42#	58.9
" 3	18.0	19.5	6.00	117.0	17000	102000	76.0	18"I55#	18"I55#	88.4
" 6	15.58	19.0	5.25	99.7	14450	75100	56.4	15"I42#	2-12"I31.5	72.0
" 10	18.0	12.5	6.00	75.0	10900	65400	49.2	15"I42#	2-12"I31.5	72.0
" 11	15.75	8.65	5.25	44.5	6450	33850	37.0	12"I35#	12"I35	38.0
Moment due to uniform load 2x20x20x155÷8						15500 / 49350				
G12	16.75	17.3	5.5	95.0	13750	76800	57.6	15"I42#	2-12"I31.5	72.0
B6	19.0	5.0	6.0	30.0	4350	27500	36.4	12"I35#	12"I35	38.0
Moment due to uniform load 3x19x19x155÷8						21000 / 48500				
G1	These girders are not symmetrically loaded and are figured by method shown under "Computations for Girders" pages 23 and 24.								18"I55	58.9
G4									12"I31.5	36.0
G5									2-12"I31.5	72.0
G7									"	"
G8									12"I31.5	36.0
G9									2-12"I31.5	72.0
G13									15"I42	58.9
B1.									12"I40	44.8

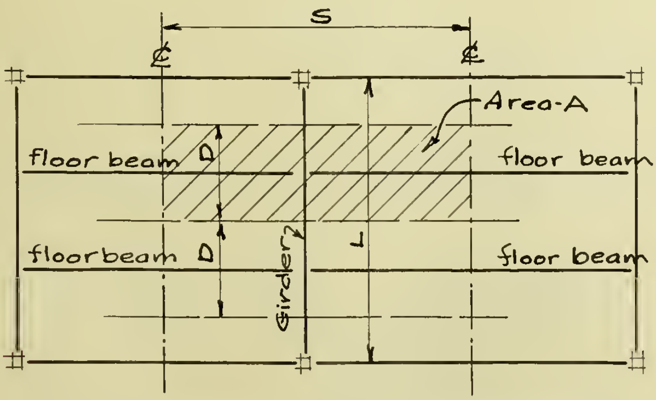


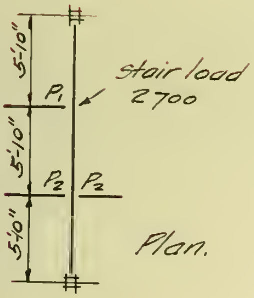
Diagram showing symbols used in table above.

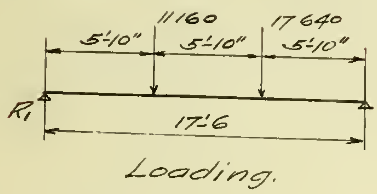
COMPUTATIONS FOR GIRDERS.

OFFICE FLOORS.

Dead load = 85; live load = 0.85 x 75 = 60; total = 145 lbs. per sq. ft.

Girder G1.





$$P_1 = 5.83 \times 10 \times 145 = 8460\#$$

$$P_2 = \text{(Reaction of B1)} = 9180.$$

$$\text{Total} = 17640.$$

$$R_2 = \frac{11.16 \times 5.83 \times 17.64 \times 11.67}{17.6} = 15\ 300\#$$

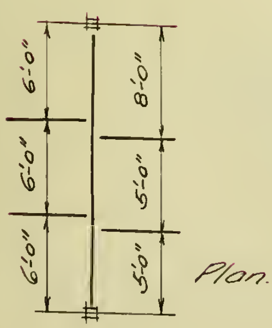
$$M = 15.3 \times 5.83 = 89\ 500\ \#''.$$

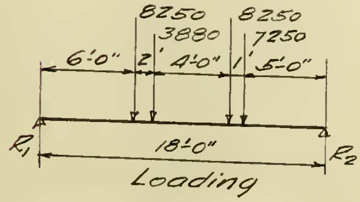
$$\frac{I}{c} = 89.5 \times \frac{12}{16.0} = 67.1$$

Least depth section useable = 15" I 55# - $\frac{I}{c}$ = 68.1.

Use - 18" I 55# - $\frac{I}{c}$ = 88.4.

Girder G5.





$$P_1 = 9.5 \times 6.0 \times 145 = 8250\#.$$

$$P_2 = 1.0 \times 2.5 \times 155 = 3880.$$

$$P_3 = 1.0 \times 5.0 \times 145 = 7250.$$

$$R_2 = \frac{8.25 \times 6 + 3.88 \times 8 + 8.25 \times 12 + 7.25 \times 13}{18.0} =$$

15 100#.

R_1 = 27.63 - 15.10 = 12.53#.

Maximum Moment at c.

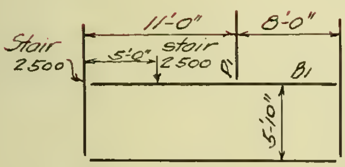
M_c = 15.1 x 6 - 7.25 x 1 = 82.35.

$\frac{I}{c}$ = 82.35 x 0.75 = 61.9.

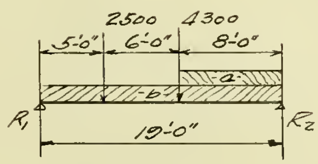
Least depth section useable = 15" I 50# - $\frac{I}{c}$ = 64.5.

Use 2 - 12" Is 31.5 - $\frac{I}{c}$ = 72.0.

Girder B1.



Plan.



Loading.

P_1 = reaction Ba = 4300#.

Mom. at center due to uniform load (b) =

$$\frac{2.92 \times 155 \times 19 \times 19}{8} = 20\ 400\#''.$$

Uniform load (a) = 2.92 x 155 x 8 = 3620.

Max. moment will occur under load L_2 (by inspection).

Moment uniform load (a) at L_2 = $\frac{4.0}{19}$ x 3620 x 11 = 8400#".

R_2 (conc. loads) = $\frac{25 \times 5.0 + 4.3 \times 11}{19}$ = 3130.

M_{L_2} (conc. loads) = 3.13 x 8 = 25 000.

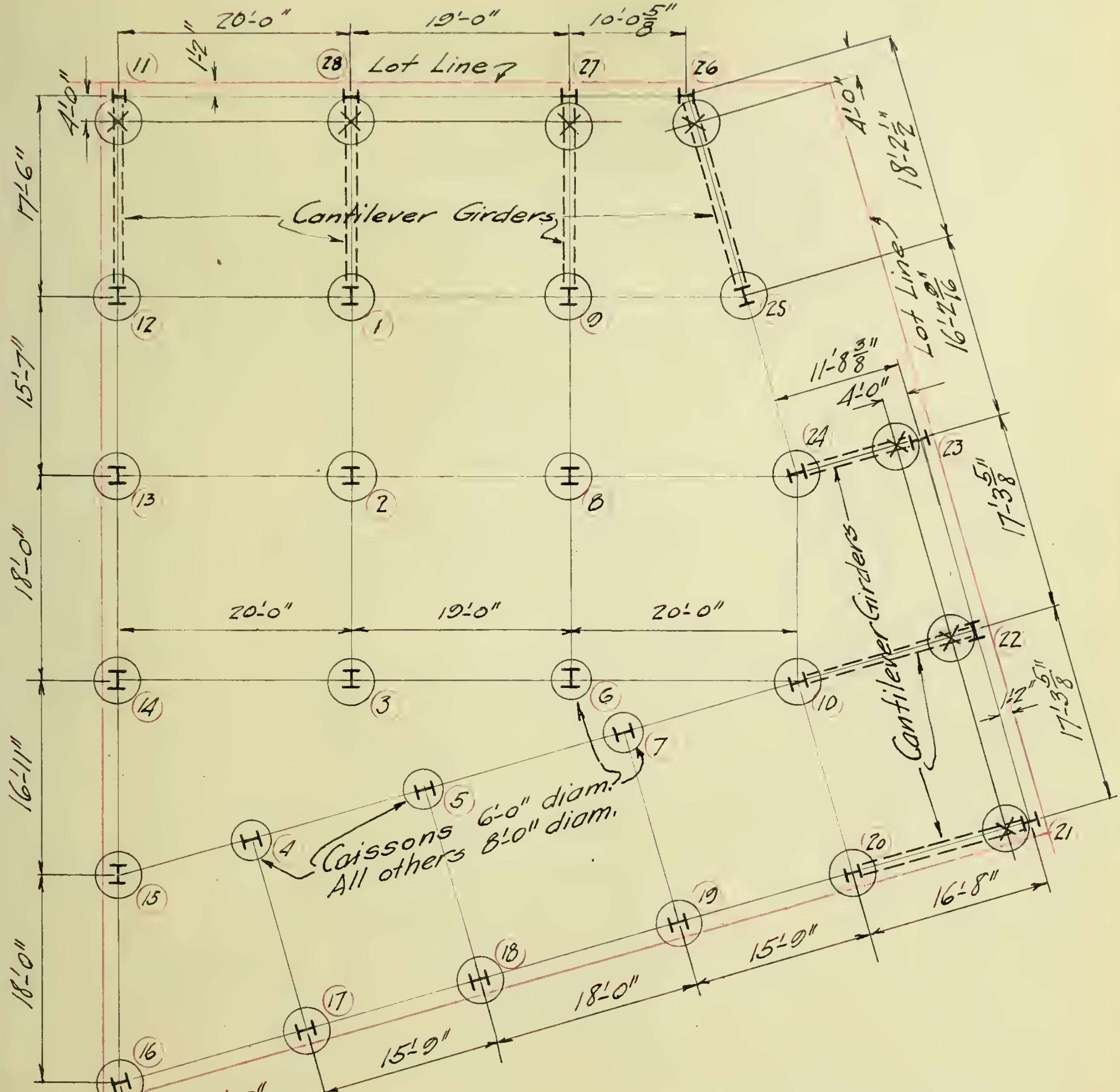
Total M_{L_2} = 20 400 + 8400 + 25 000 = 53 800 #".

$\frac{I}{c}$ = 5.38 x 0.75 = 40.4.

Least depth section useable = 12" I 40# $\frac{I}{c}$ = 41.0.

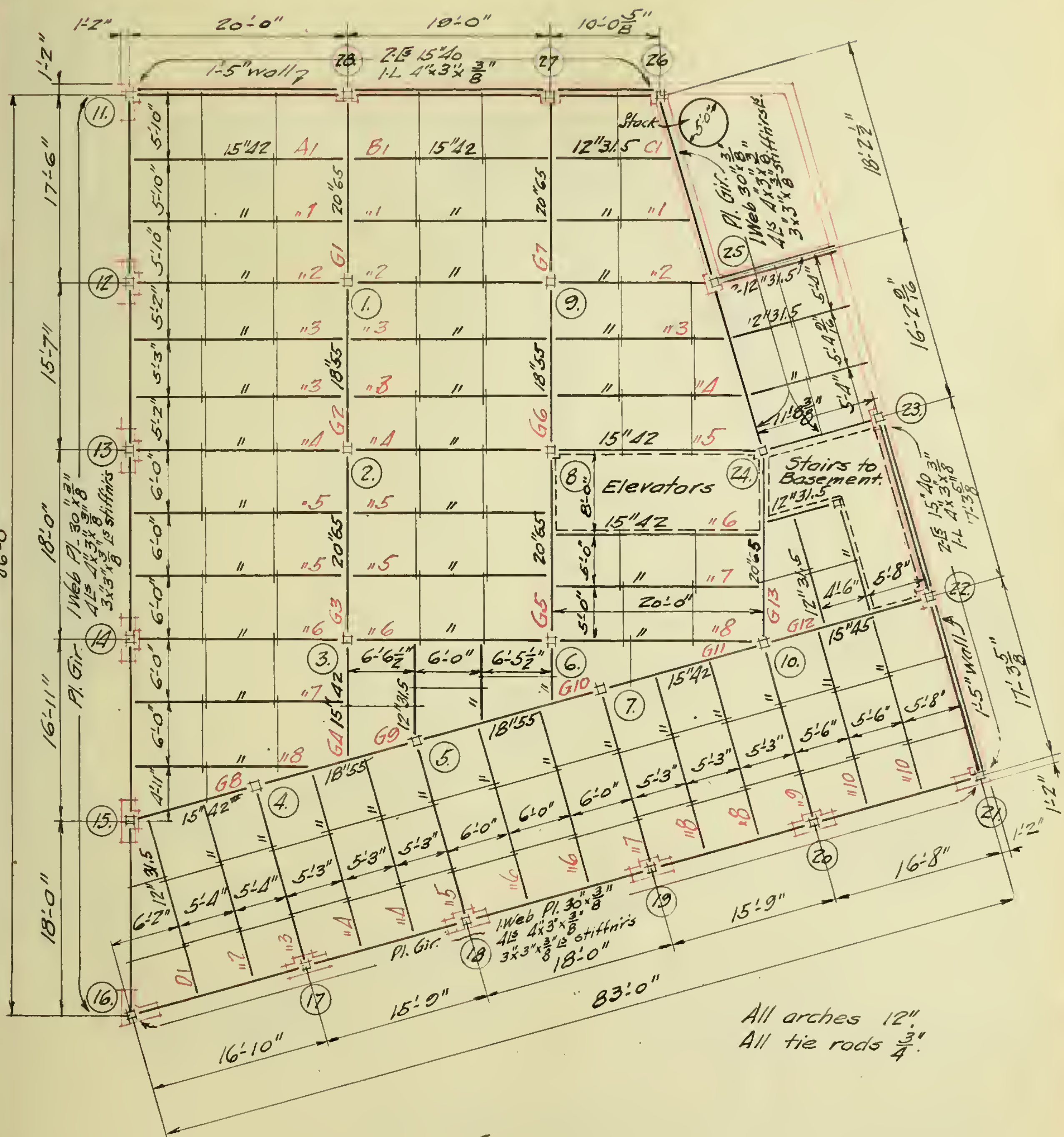
Use 12" I 40#.

PLATE-VIII.

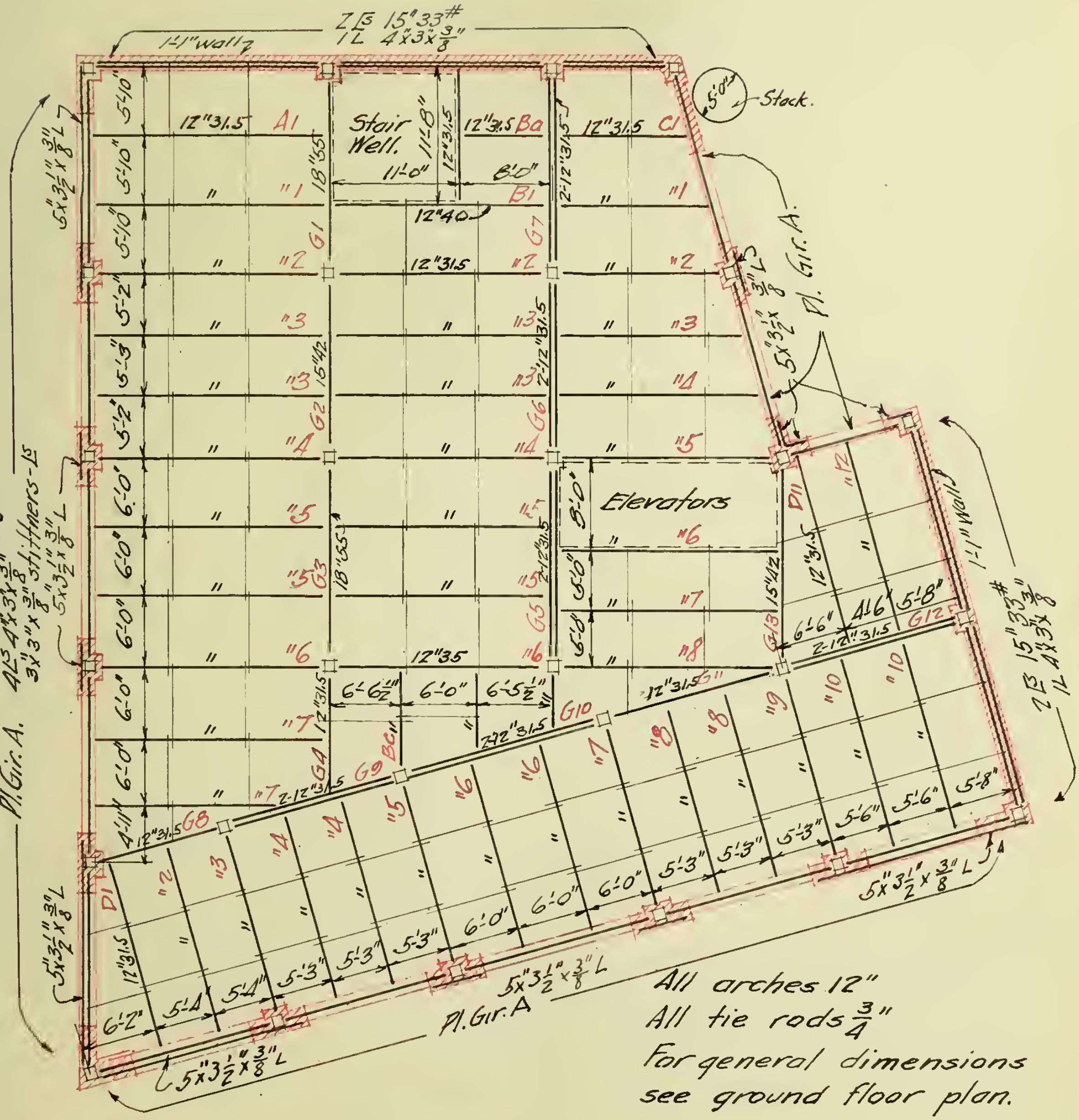


- Foundation Plan -

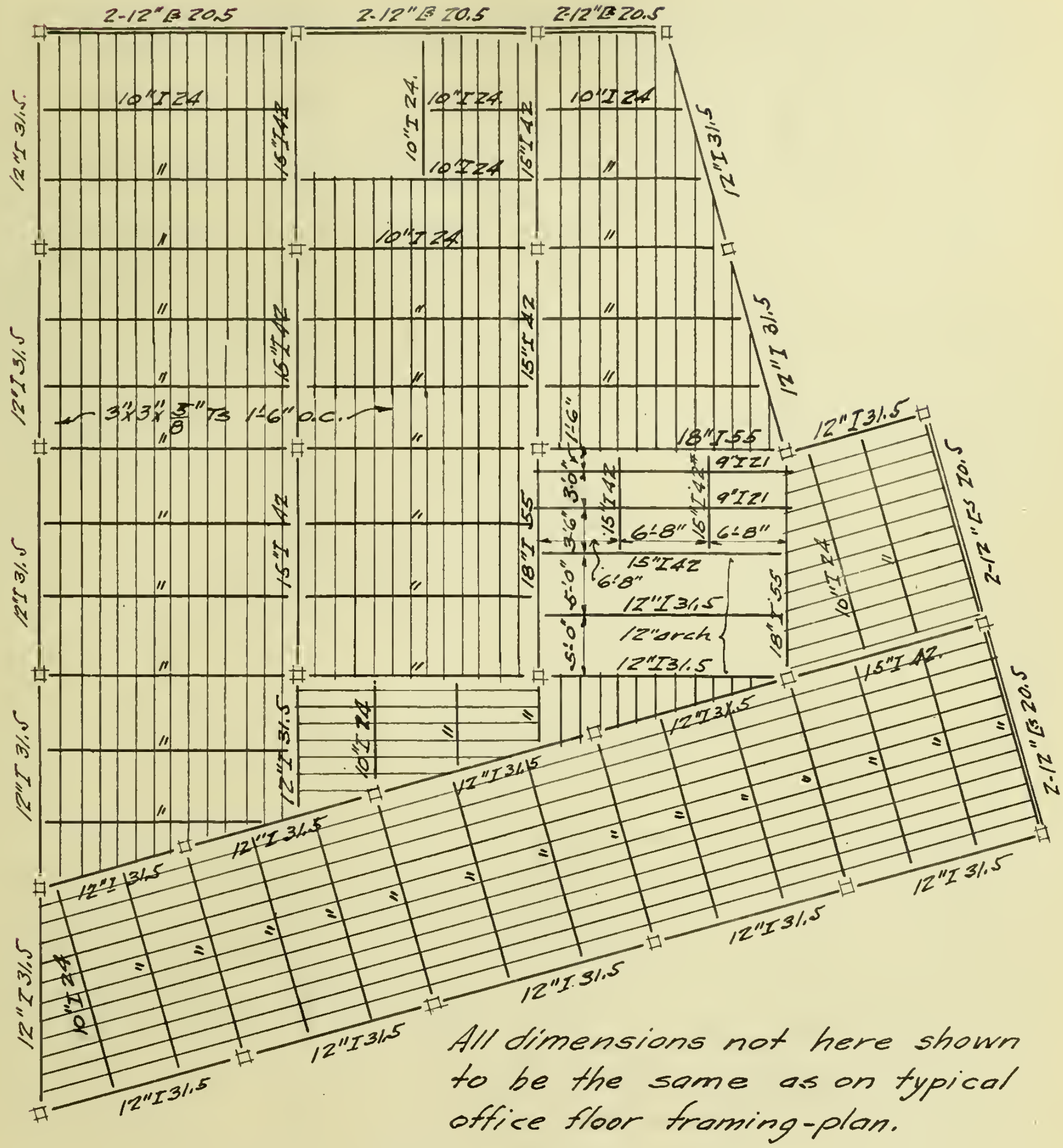
PLATE. IX.



-Ground Floor Framing-Plan-



-Typical Office Floor Framing-Plan.-



-Roof Framing-Plan-

COMPUTATIONS FOR WEIGHT OF FLOORS (see Plate VII).

Steel I-beams, 12" 35# 6'-0" o.c.	6.0	lbs./sq.	ft.	
Terra-cotta arch, 12"	35.0	"	"	"
Flooring, 3/4" maple	3.5	"	"	"
Sleepers, 3" x 3" pine 18" o.c.	0.5	"	"	"
Cinder concrete, 3"	15.0	"	"	"
Plaster, 3/4"	5.0	"	"	"
Partitions, 6" tile	20.0	"	"	"
Total for office floors	[illegible]5.0	"	"	"
Tile or mosaic floor	[illegible]4.0	"	"	"
Total for main floor	95.0	"	"	"

Weight of Roof. (see Plate VII).

Terra-cotta book-tile, 4"	22.0	lbs./sq.	ft.	
Tar and gravel composition roof	6.0	"	"	"
Steel, 3" x 3" x 3/8" Ts. 18" o.c.	5.0	"	"	"
Roof Framing 10" x 24# I	5.0			
	38.0			
say	40.0	"	"	"
Suspended ceiling	10.0	"	"	"
Total	48.0	"	"	"
say	50.0	"	"	"

ELEVATOR LOADS

The following data were used in designing the elevator framing at the roof:

Area of elevator 6'-2" x 5'-2"	31.0	sq.ft.
Dead load of cage	2200	lbs.
Live load of cage 31.0 x 75	2330	"
Counter weight and ropes	3900	"
35 H. P. motor	3350	"
Machinery	2650	"
Total	14 500	"

The computations necessary are similar to the girder computations shown on pages 23 and 24. Plate XI shows the framing over the elevator shafts, and gives the sizes and weights of the members used.

COLUMNS

The columns of this building may be divided into two groups, interior and wall columns. The interior columns carry the floor and roof loads, and some of them carry the elevator loads. The wall columns carry floor and roof loads, together with the load of the masonry wall. The load coming upon an interior column at any floor is equal to the sum of the reactions of the girders and beams framing into it. The sum of these reactions for any floor will be equal to the floor area supported by the columns multiplied by the unit dead load plus the unit live load for that floor. The floor area supported by any column can easily be determined from the spacing of the columns. Table D gives the loadings for five

interior columns. In this table the two columns on the left give the dead and live loads per square foot for each floor. At the top of the table are given the numbers of the column, which refer to their position on the floor-plan, and the number of square feet of floor area supported by the column. The numbers opposite the items give their weights in pounds for each column. The item "column" includes the weight of the column itself, together with the weight of the terra-cotta casing. This weight was approximated by summing up the loads exclusive of the weight of the column, and then approximating the column section. The assumed weight here used is consistent with the actual weights of columns and casings in similar buildings. In floors 1 to 11 inclusive, the column weight has been included under the item "dead floor load" so as to condense the table. The loads have been summed up at every second floor, Table E (similar to Table D) gives the loadings for four wall-columns. Column 18 is a typical street wall-column, concentrically loaded. Column 28 is a typical blank-wall column, also concentrically loaded. Column 25 is loaded similar to column 18, and is a column in the light-court wall. Column 11 is a typical, eccentrically loaded corner column. The table gives the weights of the masonry spandrels, cornice, piers, and sidewalk, which are carried at the various floors. To determine the weights of the spandrels, cornices, and piers, they must first be designed. In this design these weights were determined from preliminary designs, which, however, did not differ greatly from the final design (see Plates XII and XIII). The cross-section of the

TABLE-D.

COLUMN LOADINGS. FOR INTERIOR COLUMNS.

D	L′	Story	Number of col.	2	5	6	8	10
			Floor area supported	328.0	220.0	214.0	260.0	306.0
50	40	Roof	Roof load dead	16400	11000	10700	13000	15300
			" " live	13100	8800	8600	10400	12200
			Elevator load			3600	18000	3600
			Column.			2000	2000	2000
			Total.	29500	19800	24900	43400	33100
85	70	12	Floor load dead	27800	18720	18000	22100	27000
			" " live	23000	15400	15000	18200	21400
			Column.	2000	2000	2000	2000	2000
"	61	11	Floor load dead	29800	20700	20000	24100	29000
			" " live	19400	13400	13100	15900	18700
			Total	131600	90000	93000	125700	122200
"	54	10	Floor load dead	30300	21200	20500	24600	29500
			" " live	17700	11900	11600	14000	16500
"	47	9	" " dead	30300	21200	20500	24600	29500
			" " live	15400	10300	10000	12200	14400
			Total	225400	154600	155600	201100	212100
"	40	8	Floor load dead	30800	21700	21000	25100	30000
			" " live	13100	8800	8600	10400	12200
"	33	7	" " dead	30800	21700	21000	25100	30000
			" " live	10800	7300	7100	8600	10100
			Total	311000	214100	213300	270300	296400
"	26	6	Floor load-dead	31300	22200	21500	25600	30500
			" " live	8500	5700	5600	6800	7900
"	19	5	" " dead	31300	22200	21500	25600	30500
			" " live	6200	4200	4100	4900	5800
			Total.	388500	268400	266000	333200	371100
"	12	4	Floor load dead	31800	22700	22000	26100	31000
			" " live	3900	2600	2600	3100	3700
"	6	3	" " dead	31800	22700	22000	26100	31000
			" " live	2000	1300	1300	1100	1800
			Total	458100	317700	313900	389600	437600
"	2	2	Floor load dead	31800	22700	22000	26100	31000
			" " live	700	400	400	500	600
95	45	1	Floor load dead	35150	24900	24300	28700	31000
			" " live	14800	9900	9600	11700	13800
Total Dead Load.				391900				
Total Live Load				148600				
Total Dead + Live Loads				540500	375600	370200	456600	514000

TABLE-E.

COLUMN LOADINGS FOR WALL COLUMNS.

D	L′	Story	Number of Col.	18	28	25	11	
			Floor area support'd	146.0	170.0	132.0	Concentric 29.0	Eccentric 58.5
50	40	Roof	Roof load dead	7300	8500	6400	1400	2900
			" " live	5800	6800	5300	1200	2300
			Masonry pier		12700	12200	6500	21000
			Cornice	40400		10300		
			Total.	53500	28000	34200	35300	26200
85	70	12	Floor load dead	12400	14400	11200	2500	5000
			" " live	10200	11900	9200	2030	4100
			Masonry pier	5800	30400	12200	15600	5850
			Spandrel.	12700		10280		4100
			Column	2000	2000	2000	2000	
"	61	11.	Floor load dead	12400	14400	11200	2500	5000
			" " live	8900	10400	8000	1800	13600
			Masonry pier	12200	30400	12200	15600	12200
			Spandrel.	8450		10300		2700
			Column.	2000	2000	2000	2000	
			Total.	140600	143900	122780	121700	42500
"	54	10	Total for floor	48000	56500	43400	22100	23100
"	47	9	" " "	47000	55300	42400	21900	22700
			Total.	235600	255700		211500	45700
"	40	8	Total for floor	46400	54600	42000	22200	22300
"	33	7.	" " "	45400	53400	41000	22000	21800
			Total	327400	363700	291600	299800	44100
"	26	6	Total for floor	44900	52700	40600	22300	21400
"	19	5	" " "	43900	51500	39700	21100	21000
			Total	416200	467900	371900	386700	42400
"	12	4	Total for floor	43300	50800	39300	22400	20600
"	6	3	Floor load. dead	12400	14400	11200	2500	5000
			" " . live	900	1000	800	100	400
			Masonry pier.	12200	30400	12200	15600	12200
			Spandrel	12680		10300		5000
			Column	4000	4000	4000	4000	
			Total	501800	568500	449700	473500	42200
"	2	2	Floor load dead	12400	14400	14500	2500	5000
			" " live	300	300	300	100	100
			Masonry pier	13400	30400	12200	13400	
			Spandrel.	8400		10300	16600	2700
			Column.	4000	4000	4000	4000	
95	45	1	Floor load. dead	13900	16100	12500	2800	5600
			" " live	6600	7600	6000	1300	2600
			Masonry pier	20200	3000	15000	35800	
			Sidewalk-dead+live	50900			17600	17600
			Column.	4000	4000	4000	4000	
Total dead + live loads				605900	675800	528500	604200	38500

spandrel was approximated from sketches, and 130 pounds per cubic foot was used as its weight. This is greater than the weight of brick masonry, but was adopted on account of the fact that the spandrel will contain considerable steel, and must carry the weight of the window frames, sash, and glass. The corner columns will be eccentrically loaded, and must be designed accordingly. The table shows the amount of concentric and eccentric load coming upon the columns at each floor. The eccentric load is the load carried by the girder which produces bending moment in the column. The eccentric load of each floor will probably be rapidly transferred to the center of gravity of the column-section, and it would be sufficiently safe to consider only one girder reaction as eccentric in each story-length of the column. In this design, however, the sum of two girder reactions was considered as the eccentric load in a two-story length of the column. This will simplify the computations, and will be on the side of safety, without producing an extravagent section.

The column-section adopted in this design is composed of a web plate, four angles, and two or more cover plates. The back to back distance of the angles was kept constant from the basement to the roof, so as to simplify the splices and keep the lengths of the corresponding girders and beams uniform in all stories. The columns will be spliced every two stories, the splices being a few feet above the floor level.

In summing up the loads on each column, it was found that the columns could be divided into four groups whose loadings are very similar. The one having the greatest loading

of each group was designed, and the other columns of that group were made the same size. There will be six eccentrically loaded corner columns. In summing up the loads on these columns it was found that columns 11, 16, 21 and 23 have very nearly the same loadings, and the others have a somewhat smaller loading. Column 11 supports the greatest concentric and eccentric loads, and was designed for this group. Table (F) shows the column schedule for the building. At the top of the table are given the numbers of the columns included in each group and below each group is given the column-section for each two-story length from basement to roof. The final computations will be shown for four stories of columns for both concentric and eccentric loadings. The method used in these computations is "cut and try" and those here shown are the final trials.

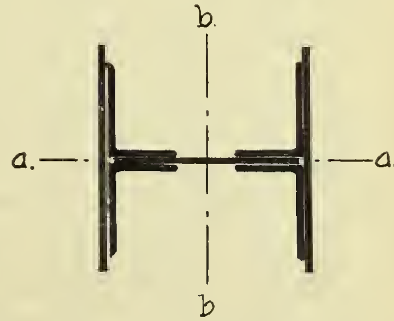
Column Computations.

Design of column 2.

Basement to 2nd floor.

Unsupported length = 15'.

Load = 540 500 lbs.



Section	Area sq. ins.	Ia - a
4 Ls 6 x 4 x $\frac{9}{16}$	21.24	185.53
1 pl. 12 x 1/2	6.00	0.13
2 pls. 14 x 5/8	17.50	285.84
Total	44.74	471.50

TABLE. F.

COLUMN SCHEDULE

Stories		Cols. No. 4, 5, 6, 7	Cols. No. 1, 2, 3, 6, 8, 9, 10.	Cols. No. 22, 27, 28, 13, 14, 15, 17, 18, 19, 20.	Cols. No. 11, 16, 21, 23, 24, 26.
Story height 12ft.	Roof 12th.	4Ls-3"x3"x3/8" 5/16" stay pls. Area. 8.44□"	4Ls-4"x3"x3/8" 5/16" stay pls. Area. 9.92□"	4Ls.4"x3"x3/8" 3/8" stay pls. Area 9.92□"	4Ls.4"x3"x3/8" 3/8" stay pls. Area. 9.92□"
" "	11th. 10th.	Web 12"x5/16" 4Ls-3"x3"x3/8" Area 8.44□"	Web. 12"x5/16" 4Ls-4"x3"x3/8" Area 13.67□"	Web 12"x3/8" 4Ls-4"x4"x3/8" Area 15.94□"	Web. 12"x3/8" 4Ls-3"x3"x3/8" 2pls 8"x3/8" Area. 18.94□"
" "	9th. 8th.	Web. 12"x3/8" 4Ls 4"x3"x3/8" Area 14.42□"	Web. 12"x3/8" 4Ls-4"x3"x3/8" 2pls. 9"x3/8" Area 21.17□"	Web 12"x3/8" 4Ls-4"x4"x3/8" 2pls. 10"x3/8" Area 23.44□"	Web 12"x3/8" 4Ls-4"x4"x3/8" 2pls. 10"x7/16" Area 24.69□"
" "	7th. 6th.	Web 12"x3/8" 4Ls 4"x3"x3/8" 2pls. 9"x3/8" Area. 21.17□"	Web 12"x1/2" 4Ls-4"x4"x1/2" 2pls. 10"x3/8" Area 28.50□"	Web 12"x1/2" 4Ls-5"x3½"x1/2" 2pls. 12"x3/8" Area 31.00□"	Web 12"x1/2" 4Ls-5"x3½"x7/16" 2pls. 12"x7/16" Area. 30.62□"
" "	5th. 4th.	Web. 12"x1/2" 4Ls-4"x4"x1/2" 2pls. 10"x3/8" Area-24.94□"	Web 12"x1/2" 4Ls-5"x3½"x1/2" 2pls. 12"x7/16" Area 32.50□"	Web 12"x1/2" 4Ls-6"x4"x1/2" 2pls-14"x7/16" Area 37.25□"	Web 12"x1/2" 4Ls-5"x3½"x5/8" 2pls. 12"x1/2" Area 37.68□"
" "	3rd. 2nd	Web. 12"x1/2" 4Ls-4"x4"x3/8" 2pls. 10"x3/8" Area 28.50□"	Web. 12"x1/2" 4Ls 6"x4"x1/2" 2pls 14"x7/16" Area 37.25□"	Web 12"x1/2" 4Ls-6"x4"x5/8" 2pls. 14"x9/16" Area 45.19□"	Web 12"x1/2" 4Ls-6"x4"x9/16" 2pls-14"x9/16" Area 42.99□"
18ft. 15ft.	1st Bas'm't.	Web 12"x1/2" 4Ls-5"x3½"x1/2" 2pls. 12"x7/16" Area 32.50□"	Web. 12"x1/2" 4Ls-6"x4"x9/16" 2pls 14"x5/8" Area 44.74□"	Web 12"x5/8" 4Ls 6"x4"x5/8" 2pls. 14"x9/16" 2pls. 14x3/8 Area. 57.19□"	Web 12"x3/4" 4Ls 6"x4"x3/4" 2pls. 14"x5/8" Area 54.26□"

$$r = \left(\frac{471.50}{44.74}\right)^{\frac{1}{2}} = 3.24$$

Allowable unit stress = $16\,000 - 70\,\frac{180}{3.24} = 12120$ lb./sq. in.

Area required = $\frac{540\,500}{12\,120} = 44.50$ sq. in.

Efficiency = $\frac{44.74}{44.50} = 1.003$

2nd. floor to 4th. floor.

Unsupported length = 12'.

Load = 458 100 lbs.

Section	Area Sq. in.	Ia - a
4 Ls 6 x 4 x 1/2	19.00	164.93
1 pl. 12 x 1/2	6.00	0.13
2 pls. 14 x 7/16	12.25	200.08
Total	37.25	365.14

$$r = \left(\frac{365.14}{37.25}\right)^{\frac{1}{2}} = 3.13$$

Allowable unit stress = $16\,000 - 70\,\frac{144}{3.13} = 12780$ lbs.

Area required = $\frac{458\,100}{12\,780} = 35.95$ sq. in.

Efficiency = $\frac{37.25}{35.95} = 103.5$

Design of column 11.

Basement to 2nd. floor.

Unsupported length = 15'.

Load - total = 604 200 lbs. - eccentric load = 33500 - eccentricity = 8".

Section	Area sq. in.	Ia - a	Ib - b
4 Ls 6 x 4 x 3/4	27.76	265.35	741.27
1 pl. 12 x 3/4	9.00	0.42	108.00
2 pls. 14 x 5/8	17.50	285.84	725.50
Total	54.26	551.61	1574.77

$$\text{least } r = \left(\frac{551.61}{54.26}\right)^{\frac{1}{2}} = 3.18''$$

Eccentric moment = 33 500 x 8 = 268 000 lb. in.

$$\text{Allowable unit stress} = 16000 - 70\,\frac{180}{3.18} - 0.75 \times \frac{268\ 000 \times 6.75}{1574.77}$$

$$= 11\ 170 \text{ lbs./sq. in.}$$

$$\text{Area required} = \frac{604\ 200}{11\ 170} = 54.20 \text{ sq. in.}$$

$$\text{Efficiency} = \frac{54.26}{54.20} = 1.000.$$

2nd. floor to 4th. floor.

Unsupported length 12'.

Load - total = 473 500 lbs., eccentric = 42 200 lbs. - eccentricity = 8".

Section	Area sq. in.	Ia - a	Ib - b
4 Ls 6 x 4 x 9/16	21.24	185.53	583.35
1 pl. 12 x 1/2	6.00	0.13	72.00
2 pls. 14 x 9/16	15.75	257.26	646.80
Total	42.99	442.92	1302.15

$$\text{least } r = \left(\frac{442.92}{42.99}\right)^{\frac{1}{2}} = 3.21.$$

Eccentric moment = 42 200 x 8 = 337 600

Allowable unit stress =

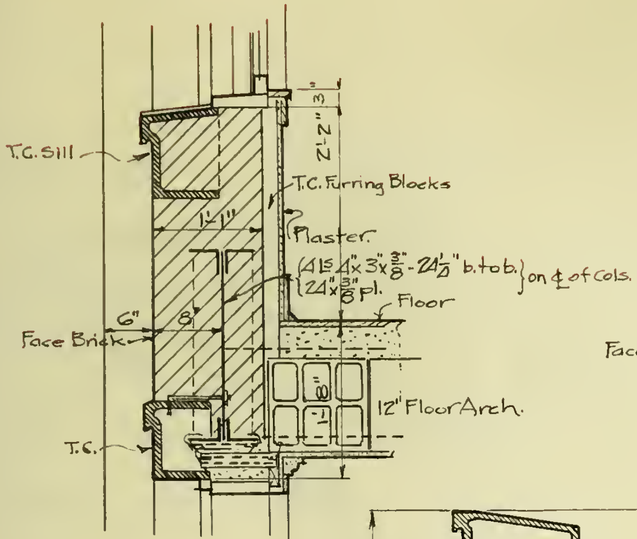
$$= 16000 - 70\,\frac{144}{3.21} - 0.75\,\frac{337\ 600 \times 6.69}{1302.15} = 11570 \text{ lbs./sq. in.}$$

$$\text{Required area } \frac{473\ 500}{11\ 570} = 40.96$$

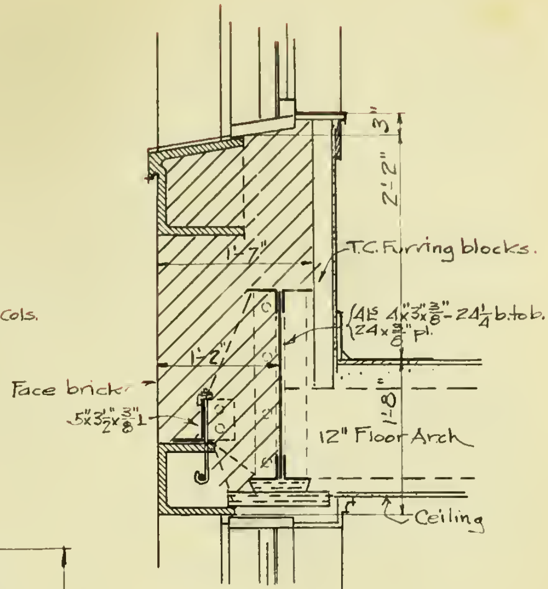
$$\text{Efficiency} = \frac{42.99}{40.96} = 104.9$$

SPANDRELS AND CORNICES

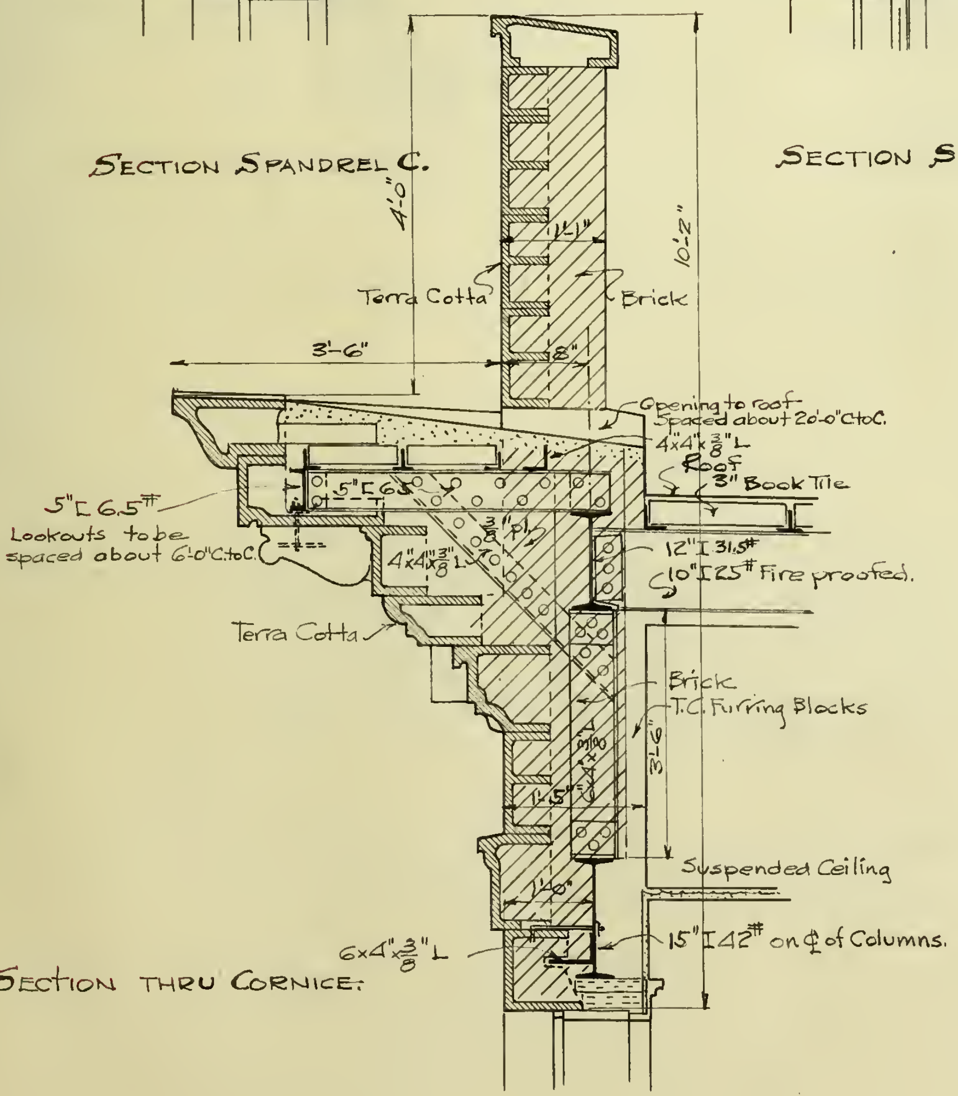
Plates (XII) and (XIII) show the typical spandrel-sections and cornice-section, as designed for this building. The spandrel beam consists of a plate girder, composed of four angles and web plate, which carries both the masonry loads and the ends of the floor beams. The drawings show the method of supporting the brick wall at the girders and columns. Where a seventeen-inch wall is used the face bricks are supported by an angle, which is framed out from the girder by means of a plate attached to the stiffener angles of the girder. In many buildings two channels or I-beams are used as spandrel beams. In this design the plate girder was used because it will have a greater depth than a beam composed of two standard shapes and will not have a great excess of section. A deep girder will afford a long riveted connection to the columns and thus make a very rigid joint. In many build-



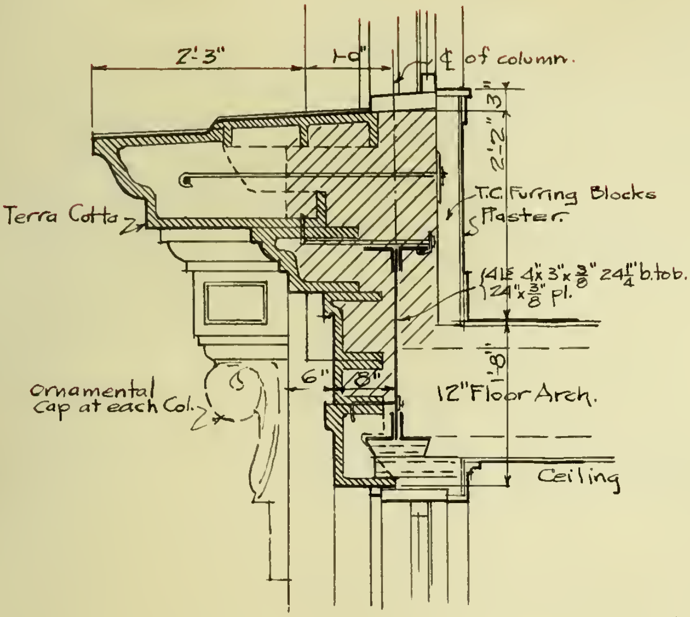
SECTION SPANDREL C.



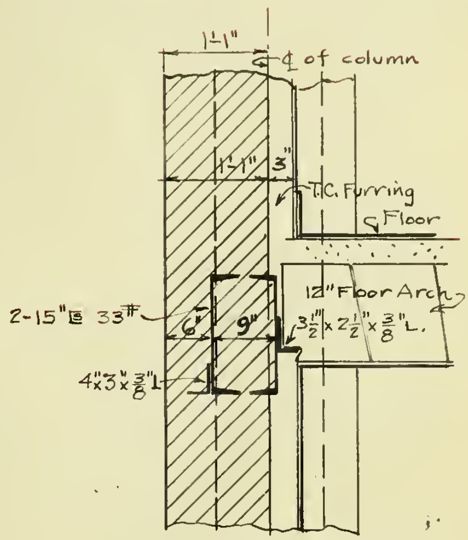
SECTION SPANDREL C!



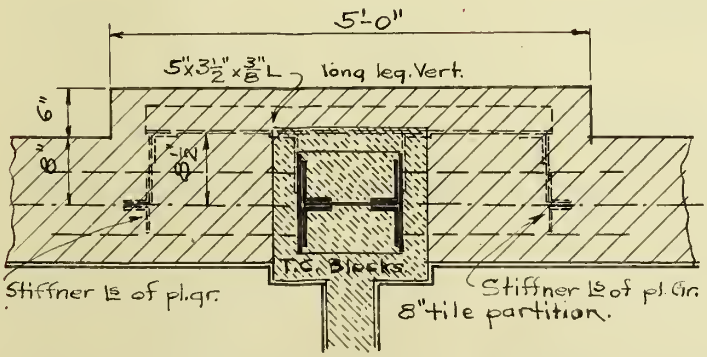
SECTION THRU CORNICE.



SECTION SPANDREL B



SECTION THRU BLANK WALL
SHOWING SPANDREL BEAM.



PLAN SECTION THRU COLUMN. (STREET ELEV.)
SHOWING METHOD OF SUPPORTING MASONRY.

ings deep girders alone are relied upon to produce sufficient rigidity to withstand the stresses due to the wind pressure. The computations will be shown for the plate girder spandrel beam C, for the spandrel beam in the blank walls, for the girder at the ground floor in the street walls, and for the beams in the cornice-section.

Computations.

Spandrel C.

Span 18 ft. (maximum)

Total weight of masonry = 1.42 x 4 x 120 x 18

= 12 200 lbs.

Floor beam reaction = 10 x 6 x 155

= 9300 lbs.

This load will be concentrated at the third points.

Moment due to masonry load = $\frac{12\ 200 \times 12 \times 18}{8}$

= 330 000 lb. in.

Moment due to concentrated loads = $\frac{8700 \times 6 \times 12}{0}$

= 670 000 lb. ft.

Total moment = 1 000 000 lb. in.

Use 24-1/4 in. b. to b. distance of angles.

Required I = $\frac{1\ 000\ 000 \times 12.12}{16\ 000}$ = 757.0

3/8 in. web plate will be used.

4-Ls 4" x 3" x 2/8" will be used because a width of at least 8 in. is required to form a proper support

of the masonry.

I of 1 pl. 24 x 3/8 = 432.0

I of 4-Ls 4' x 3 x 3/8 - 24-1/4" b. to b. 1115.0

Total 1547.0

This section will be somewhat in excess of that required, but will be used on account of the additional strength required for wind stresses.

Spandrel in Blank Wall at Office Floors.

Span = 20 ft. (maximum)

Total masonry load = 1.08 x 12 x 20 x 120

= 31 200 lbs.

Moment due to masonry load $= \dfrac{31\ 200 \times 20 \times 12}{8}$

= 936 000 lb. in.

Moment due to uniform floor load $= \dfrac{3 \times 20^2 \times 185 \times 12}{8}$

= 333 000 lb. in.

Total moment = 1 269 000 lb. in.

$$\frac{I}{c} = \frac{1\ 269\ 000}{16\ 000} = 79.4$$

Use 2 [s

$$\frac{79.4}{2} = 3.97 = \frac{I}{c} \text{ required for each [}$$

Use 2 - 15" [s 33 lbs. $\frac{I}{c} = 41.7$

For main floor use 2 - 15" [s 40 lbs. (Computations similar to those above)

Girder at Ground Floor in Street Walls.

This girder carries the ends of the floor-beams and side-walk beams.

Span = 18 ft. (maximum)

Sidewalk load = 6 x 10 x 320 = 19 200 lb.

Floor load = 6 x 10 x 180 = 10 800 lb.

Total = 30 000 lb.

This load will be concentrated at third points.

Moment = 30 000 x 6 x 12 = 2 160 000 lb. in.

Use 30-1/4 in. b. to b. distance of angles.

Assume 28" as effective depth.

$$\frac{2\,160\,000}{28 \times 16\,000} = 4.82 \text{ sq. in. required area of flange.}$$

1/8 of area of web considered as concentrated in flange.

1/8 x 30 x 3/8 = 1.40 sq. in.

4.82 - 1.40 = 3.41 required area in angles.

3.41 ÷ 2 = 1.71 required area in one angle.

Area of 4 x 3 x 3/8 L = 2.11 sq. in. (1 hole out)

30.25 - (2 x 1.28) = 27.69 in. effective depth.

$$\frac{2\,160\,000}{27.69 \times 16000} = 4.87 \text{ sq. in. required flange area.}$$

$$\frac{4.87 - 1.40}{2} = 1.73 \text{ sq. in. required area one angle.}$$

Use 4-Ls 4" x 3" x 3/8" 30" x 3/8" web.

Cornice Beams.

Lower Beam. This beam carries only the weight of the

lower and projecting part of the cornice.

Area of section (approximated from drawings)

= 13.5 sq. ft.

Span = 18 ft. (maximum)

Weight of cornice = 13.5 x 120

= 1 620 lbs./ ft.

$$M. = \frac{1620 \times 18 \times 18 \times 12}{8} = 790\ 000 \text{ lb. in.}$$

$$\frac{I}{c} = \frac{790\ 000}{16\ 000} = 49.3$$

Use 15" I 42# $\frac{I}{c} = 58.9$.

Upper Beam. This beam carries the end of the roof beams, together with the weight of wall above the cornice.

Total weight of masonry = 1.08 x 5.0 x 115 x 8 = 11 200#.

$$\text{Moment due to masonry load} = \frac{11200 \times 18 \times 12}{8}$$

= 302 000 lb. in.

Roof load = 84 x 6 x 10 = 5 040#.

This load will be concentrated at the third points.

Moment due to roof load = 5040 x 6 x 12

= 36 200

Total M. = 338 200 lb. in.

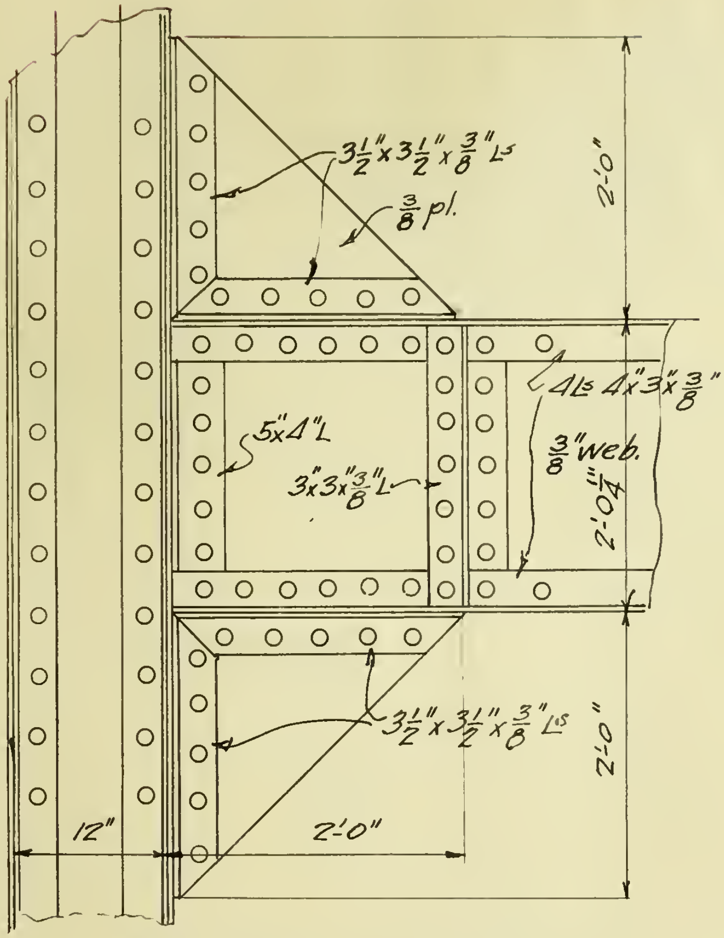
$$\frac{I}{c} = \frac{338\ 200}{16\ 000} = 22.6 .$$

10" I 25# $\frac{I}{c} = 24.4$.

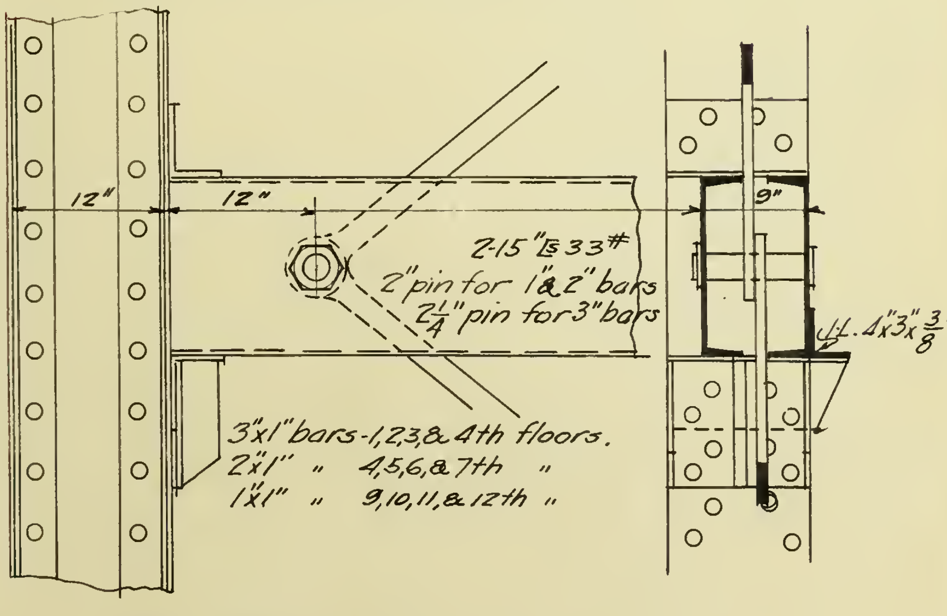
Use 12" I 31.5 $\frac{I}{c} = 36$.

WIND BRACING

The wind acting upon the exposed surface of the building will produce stresses in the steel framework, for which provision must be made. The amount of wind bracing necessary will depend upon the proportions of the building. There are some engineers who claim that wind pressure need not be considered when the height of the building does not exceed two and one-half times its least width at the base. In the New York Ordinance no special consideration of wind stress is prescribed for buildings, which have an exposed height of four times or less the least dimension of the base of the building. The Chicago Ordinance states that allowance shall be made for wind pressure in buildings whose heights exceed one and one-half times their least horizontal dimensions. In this building the height is equal to about twice the average horizontal dimensions, and, therefore, wind bracing must be provided. The wind bracing commonly used consists of bracing in a vertical plane between columns. The bracing is either in the form of diagonal members or knee-braces are placed between the columns and the horizontal members. In some buildings single deep guiders are used at the floor level in the walls between columns. Diagonal members can not often be used on account of interferring with wall openings. They are sometimes used in interior partitions or in exterior walls in which there are no window openings. Diagonal bracing will be used between the columns in the blank walls of this building. Knee-braces will be used at the girder and column con-



Wind Bracing - At girders in street and light-court walls.



Wind Bracing - At girders in blank walls.

nections in the other exterior walls. The diagonal bracing will consist of eye bars varying from a three-square-inch-section at the lower floors to a one-square-inch-section on the upper floors. The knee-bracing will consist of a gusset plate, riveted to the girder angles and to the columns. Plate (XIV) shows the detail of the knee bracing and of the connection of the diagonal members.

This completes the general design of the steel superstructure. The design of the foundations and cantilever girders is necessary to fully complete the structural design of the building, but, as before stated, they will not be included in this thesis.

Finis.